Modelling thermal profiles within protected concrete filled hollow steel sections

Modelling Thermal Profiles in Fire-Exposed Concrete Filled Hollow Steel Sections Protected with Intumescent Paint

DAVID RUSH[1], LUKE BISBY[1] and ALLAN JOWSEY[2]

ABSTRACT

Concrete filled hollow steel (CFS) sections are increasingly used in multi-storey buildings and can, in some cases, provide adequate fire resistance without the need for supplemental fire protection. When calculations show that unprotected CFS sections have inadequate structural capacity at the required fire resistance period (FR), these sections require applied fire protection. For plain steel sections (i.e. unfilled hollow steel sections) the design structural capacity at the required FR is typically assumed to depend only on the limiting steel temperature, and is assumed to be the same for both unprotected and protected sections; however, for CFS sections this assumption does not apply. Heated CFS sections develop thermal gradients within their concrete cores. The cross-sectional thermal gradients, upon which the material properties and structural capacities of CFS sections depend, are influenced by many factors such as: the limiting steel tube temperature assumed in design; the required fire resistance period; and the cross-sectional dimensions. This paper presents a parametric analysis on the effect of these three key parameters on the thermal gradients within CFS sections exposed to an assumed steel temperature-time history. This is performed using ABAQUS finite element (FE) modelling. This is used to develop a simplified design equation to predict the thermal gradients within CFS columns based on these parameters, supporting a rational, simplified approach for fire resistance prediction of protected CFS columns.

INTRODUCTION

Architects and engineers increasingly specify concrete-filled steel hollow structural sections (CFS) in the design and construction of multi-storey buildings, as these are an attractive, efficient, and sustainable means by which to design and construct highly optimized compressive members. A CFS section consists of a hollow steel section filled with concrete and provides superior load carrying capacity and fire resistance compared with either an unfilled steel tube or a plain reinforced concrete section. The concrete infill and the steel tube work together, at

[1]BRE Centre for Fire Safety Engineering, University of Edinburgh, EH9 3JL, UK
[2]International Paint Ltd, AkzoNobel, Newcastle, NE10 0JY, UK

both ambient temperatures and during fire, yielding several benefits: the steel tube acts as stay-in-place formwork during casting of the concrete and provides a smooth, rugged, architectural surface finish; the concrete infill enhances the steel tube's resistance to local buckling; and the steel tube sheds axial load to the concrete core when heated during a fire, enhancing the columns fire resistance [1].

Multi-storey buildings often require structural fire resistance ratings of 60 minutes or more [2], which CFS sections can provide without the need for applied fire protection in many cases. However, when structural fire design requirements [1], [3], [4] show that adequate fire resistance cannot be achieved without protection, external fire protection is applied to the steel tube; intumescent coatings are the preferred fire protection option in many jurisdictions.

In practice, the design/specification of intumescent fire protection systems for CFS sections typically requires; 1) an assumed (often prescribed) limiting steel temperature (i.e. the temperature of the steel tube at which the CFS column is presumed to fail under load during a fire); 2) an effective section factor; and 3) a predefined (also normally prescribed) required period of standard fire exposure. Whilst guidance exists for both effective section factor calculations [5] and prescription of fire resistance times [2], no guidance exists on the prescription of limiting steel temperatures for CFS sections. In practice a limiting value between 520^{o}C and 550^{o}C is often assumed for the steel tube, however the conservatism of this assumption remains unknown.

For unfilled steel sections, designers are able to rationally calculate accurate limiting temperatures based on the applied load level during fire [6]. However, CFS sections experience a complex heating-rate-dependent thermal gradient within their concrete core due to the presence of the load carrying concrete mass, and thus the calculation of the limiting steel temperature for CFS sections is more complex.

Unprotected CFS columns experience steep thermal gradients in the concrete core when exposed to standard fires, and at the failure point (i.e. when the limiting temperature is reached) the steel is generally at a much higher temperature than the bulk of the concrete core. This means that the concrete core retains a large proportion of its strength for a given steel tube critical temperature. When fire protection is added, however, the heating rate of the steel tube is reduced, and thus the thermal gradient within the concrete core becomes shallower and the temperature difference between the steel tube and the concrete core temperatures is less extreme. Thus, for the same steel tube temperature the concrete core is expected to be at a comparatively higher temperature in a protected section, and thus will have comparatively less strength as compared with a similar unprotected CFS section; this is illustrated schematically in Figure 1. A protected CFS section will have a lower limiting temperature than an otherwise identical unprotected one.

Thermal gradients within protected CFS sections are therefore dependent upon the heating rate that the steel tube experiences, which in turn is mostly affected by:

1. the limiting temperature to which the steel is protected, since higher limiting temperatures are expected to lead to more severe thermal gradients in the core;
2. the required fire resistance period, with longer fire resistances producing shallower thermal gradients in the core;

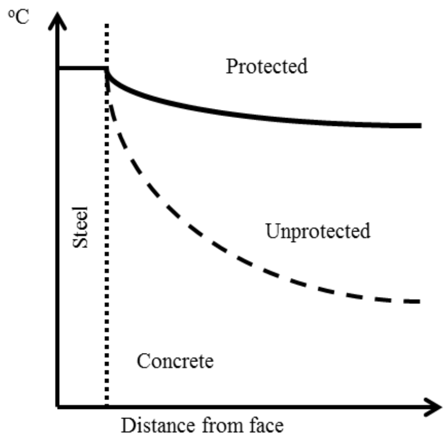


Figure 1. Comparison of thermal gradients in *Unprotected* and *Protected* CFS sections in fire.

3. the size of the CFS cross section, with larger cross sections resulting in steeper thermal gradients in the core; and
4. the thermal performance of the protection system with increasing fire exposure.

To determine the limiting steel temperature an iterative process is required, since it is effectively a function of itself, and thus for design it is desirable to have a simple, quick and conservative method to calculate limiting steel temperature for protected CFS sections. This paper assesses the thermal profiles within intumescent protected CFS sections by thermal finite element modelling, and provides a simple design basis for predicting the thermal gradients within the concrete core, from which simplified design recommendations can be developed in the future.

MODELLING

The finite element (FE) modelling was conducted using the commercial finite element software package ABAQUS and aims to understand the influence of the three key parameters on the thermal profiles within the CFS sections, namely: the size of the CFS section's concrete core, b_i; the limiting temperature of the steel, $\theta_{s,cr}$; and the required (prescribed) fire resistance period, t_{FR}. Each of these parameters was varied independently, whilst the other two parameters were held constant. Table I shows the variations in the parameters assessed within the modelling, with the constant values highlighted in grey; this resulted in 19 different finite element models being created and interrogated.

Each finite element model consisted of applying a prescribed time-temperature history (shown in Figure 2a), to the exterior nodes of the steel tube of a suitably discretized CFS section (Region A in Figure 2b), and calculating the thermal profile within the core (regions B-D in Figure 2b)) at the end of the required fire resistance period. Figure 2a shows how the variations of $\theta_{s,cr}$ and t_{FR} affect the time-temperature history, while Figure 2b shows how b_i affects the finite element mesh.

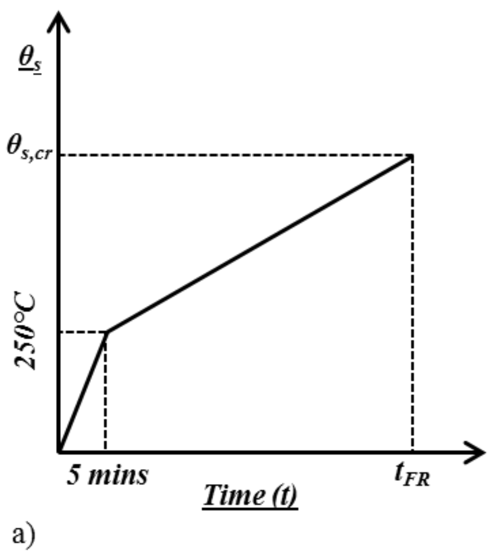
Temperatures were observed at the nodes of the 2D brick elements used, with 5 elements in Region B, 15 elements in Region C, and 9 elements in Region D, resulting in 30 nodal temperatures per thermal profile; based on a mesh sensitivity analysis [7]. An initial temperature of 20°C was assumed at every node.

Table I. Variations in the assessed parameter spaces, b_i, $\theta_{s,cr}$, and t_{FR}.

Concrete core size, b_i, (mm) *[Section size]*	Limiting steel temperature, $\theta_{s,cr}$, (°C)	Fire resistance period, t_{FR}, (mins)
148.3 *[168.3 x 10]*	300	15
199.1 *[219.1 x 10]*	400	30
253.0 *[273.0 x 10]*	500	45
303.9 *[323.9 x 10]*	520	60
386.4 *[406.4 x 10]*	600	90
437.0 *[457.0 x 10]*	700	120
488.0 *[508.0 x 10]*	800	180

The time-temperature heating curve (Figure 2a) for the steel was assumed to be bi-linear, with an initial rapid heating stage (pre-intumescent) followed by a reduced heating rate (post-intumescent) up to a maximum of the steel limiting temperature, $\theta_{s,cr}$, at the required fire resistance time, t_{FR}. Figure 2a shows an approximation of the steel tube temperature profiles that were observed during furnace testing, also by the authors, of intumescent-coated CFS sections exposed to standard fire curves [5]. Figure 2a is based on a specific International Paint Ltd. intumescent coating and therefore is not applicable to other manufacturers' products.

Figure 3 shows the predicted thermal profiles for the 19 ABAQUS models that were created, and shows temperatures within the concrete core with respect to the relative position within a quarter cross-section (i.e. $r^* = X/(½\cdot b_i)$). Figure 3a shows the thermal profiles within the concrete core whilst varying the size of the core, b_i, and maintaining the steel limiting temperature, $\theta_{s,cr}$, and the fire resistance period, t_{FR}, at 520°C and 90 minutes, respectively. Figure 3a shows intuitively that larger sections experience steeper thermal gradients than smaller sections when exposed to the same steel time-temperature curve (Figure 2a).



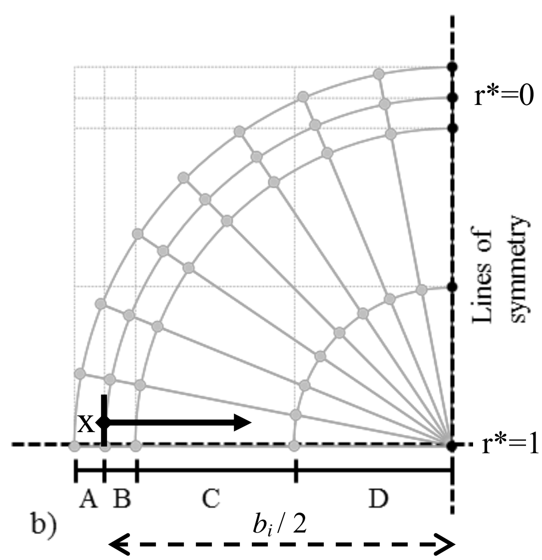


Figure 2. Schematic of (a) the idealised steel time-temperature profile; and (b) the FE model.

Figure 3b varies the steel limiting temperature, $\theta_{s,cr}$, with b_i and t_{FR} held constant at 304 mm and 90 minutes, respectively; it confirms that as the limiting steel temperature increases, for the same fire resistance period and section size, the thermal gradients become steeper. Figure 3c varies the fire resistance period, t_{FR}, with b_i and $\theta_{s,cr}$ held constant at 304 mm and 520°C, respectively; this shows that as the fire resistance period increases, for the same section and steel limiting temperature, the thermal gradients become shallower.

Figure 3 therefore shows intuitive trends that were expected and that confirm the postulations stated above: as the size and steel limiting temperature increase, the thermal gradient increases; as the fire resistance increases the gradient decreases.

ANALYSIS

In design environments where FE modelling ability may be limited and overly time consuming, it may not be practical to perform iterative finite element modelling exercises to determine the appropriate steel limiting temperature, and thus thermal profile, for every CFS section within a structure. Instead, it is desirable to have access to a rapid, simple, conservative design equation upon which to rely.

In a semi-infinite solid, the thermal profile under steadily increasing transient heating will take an exponential form. For a wall or axisymmetric section the thermal profile can be idealised as a combination of two semi-infinite solids. The hyperbolic cosine function in Figure 4a represents such a combination, given by:

$$\cosh(x) = \frac{e^{x} + e^{-x}}{2} \tag{1}$$

To create a design equation, the thermal profiles calculated from the FE modelling were fit to Equation (1) using three coefficients a, b, and c in the form:

$$\theta_c = a \cdot \cosh\left(\frac{{x^*}^{c}}{b}\right) \tag{2}$$

where $x^* = r^* - 1$, and θ_c is the temperature of the concrete at any given location.

Equation (2) was fit (grey lines in Figure 3) to all 19 of the thermal profiles calculated from the FE study, and the coefficients a, b, and c were determined using the GRG nonlinear solver in Microsoft Excel for smooth functions. The maximum error between the FE modelling and Equation (2) was 11°C (Figure 3c, t_{FR} = 15 mins) and the average error for the 19 thermal models was typically less than 2°C.

Figures 4b, 4c and 4d show the coefficients a, b and c, determined for Equation (2) with respect to the variations in concrete core size, b_i, steel limiting temperature, $\theta_{s,cr}$, and period of fire resistance, t_{FR}, respectively. It was determined that coefficient a was equal to the temperature found at the centre of the cross-section. This is because when r^* = 1 then x^* = 0 and thus the *cosh* function equals 1. Coefficient a takes an exponential function form when considering the effects of the cross-section size (Figure 4b), a linear function form when considering the effects of limiting steel temperature (Figure 4c), and a cubic function form when considering the effects of the period of fire resistance (Figure 4d).

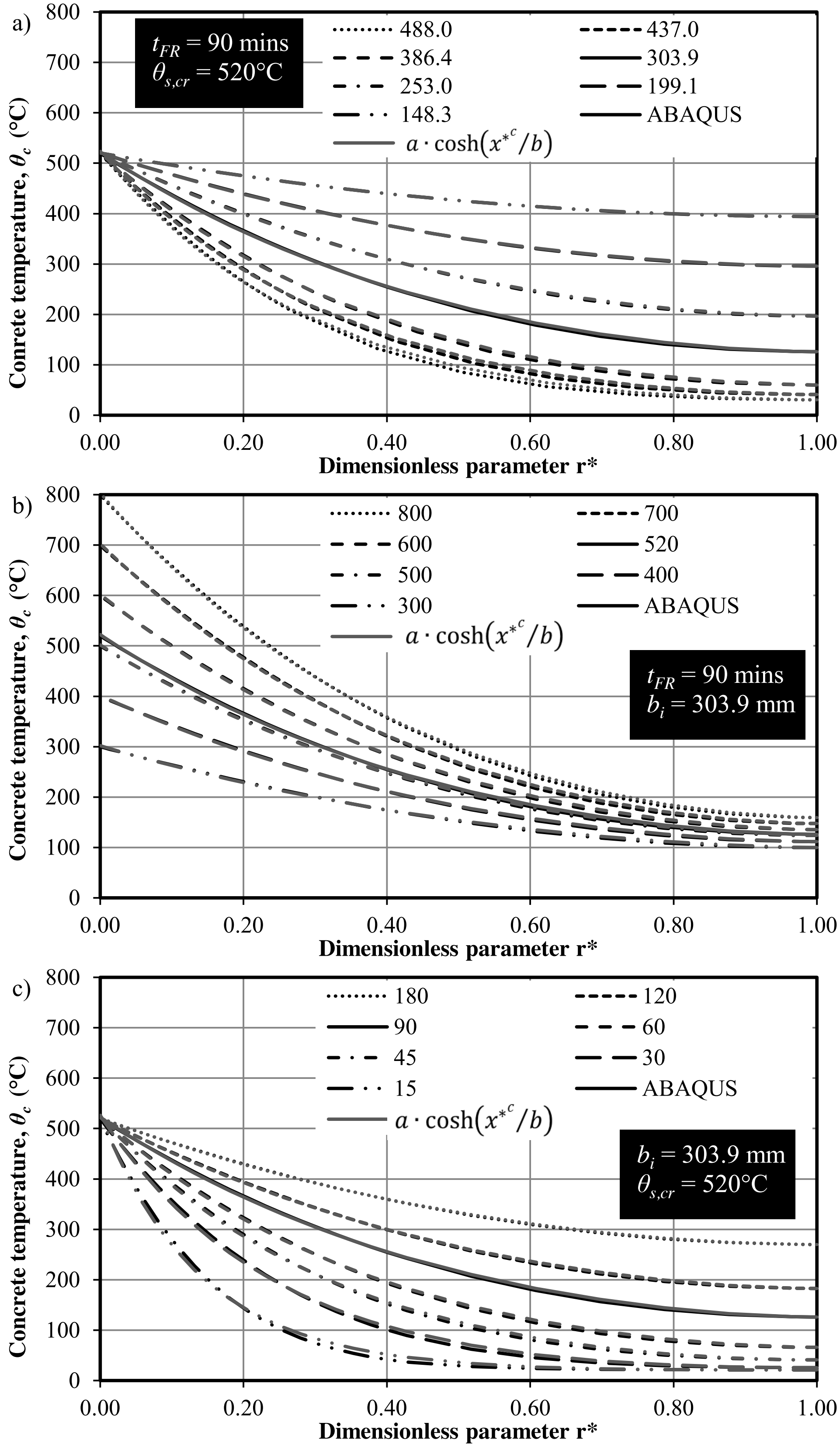


Figure 3. Concrete core thermal profiles modelled using FE and the COSH(x) function used herein when varying (a) concrete core size, b_i; (b) limiting steel temperature, $\theta_{s,cr}$; and (c) fire resistance period, t_{FR}.

These functional forms for coefficient *a* are as expected; as the concrete core size increases, its effect on the centreline temperatures decreases due to the concrete core tending toward representing a semi-infinite solid, and thus centreline temperatures of 20°C. The simplified material models used herein cause the steel limiting temperature to act as a scalar of the centreline temperature. The relative change in the gradient of the time-temperature profile after 5 minutes, combined with the time lag of the temperature wave, produces the cubic function seen for coefficient *a* in Figure 4d.

The forms of the functions that define coefficients *b* and *c* are more difficult to reason intuitively; however it is apparent that coefficient *b* takes the form of an inverse power function when considering the effects of both the cross-section size (Figure 4b), and the limiting steel temperature (Figure 4c), and a cubic function when considering the effects of the period of fire resistance (Figure 4d). Coefficient *c* also appears to take the form of a cubic function when considering the effects of both the cross-section size (Figure 4b), and the limiting steel temperature (Figure 4c); however the form of the function for coefficient *c* for the effects the period of fire resistance (Figure 4d) is less obvious.

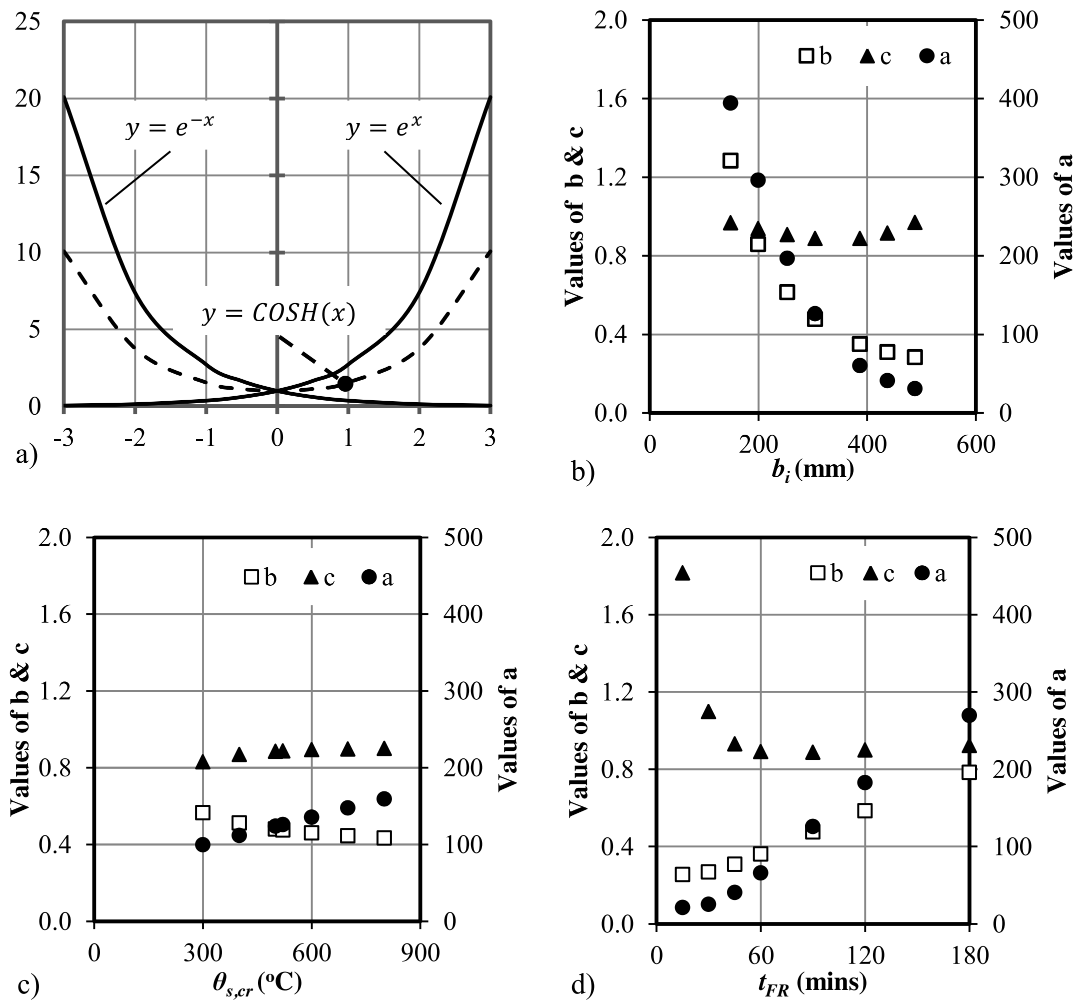


Figure 4. (a) Graphical description of the COSH(x) function; and variations in coefficients a, b, and c, with respect to: (b) concrete core breadth, b_i; (c) limiting steel temperature, $\theta_{s,cr}$; and (d) fire resistance period, t_{FR}.

Whilst the equations to determine coefficients a, b and c, are not yet fully developed, this study has shown that it is possible to accurately model the thermal profiles within protected CFS cross-sections. Further analytical investigation is required to understand the forms of coefficients a, b and c, and how they combine across the different parameters varied within this study. In the future this will lead to the development of simplified design equations for calculating the thermal profiles within protected CFS sections, and thus for the calculation of appropriate design limiting steel temperatures and fire resistances.

CONCLUSIONS

This paper has presented a series of FE modelling results of concrete filled hollow (CFS) sections subjected to an assumed time-temperature history that notionally represents the steel temperatures experienced when CFS sections are protected with intumescent coatings [5]. The thermal profiles within the CFS sections were modelled using simplified material thermal propertied from Eurocode 4 [4] form which it can be concluded that:

- as the section size increases the thermal gradient also increases non-linearly;
- as the limiting steel temperature increases the thermal gradient increases linearly;
- as the fire resistance period increases the thermal gradient decreases non-linearly; and

The paper has also presented a framework for a simplified design equation to calculate the thermal profiles within protected CFS sections in the form of a hyperbolic cosine function with three coefficients that depend on a combination of concrete core size, period of fire resistance, and limiting steel temperature. While the final simplified design equation has yet to be determined, this paper has shown that the hyperbolic cosine function can be used accurately to define the thermal profiles. Additional work is required to develop design equations and recommendations.

REFERENCES

[1] V. Kodur, "Guidelines for fire resistant design of concrete filled steel HSS columns: State-of-the-art and research needs," *Int. J. Steel Struct.*, vol. 7, no. 3, pp. 173–182, 2007.

[2] Communities and Local Government, "Approved Document B – Volume 2 – Buildings other than dwelling houses," London, UK, 2007.

[3] T. Lennon, D. B. Moore, Y. C. Wang, and C. G. Bailey, *Designers' guides to the Eurocodes*. London, UK: Thomas Telford Publishing, 2007.

[4] CEN, "BS EN 1994-1-2:2005: Eurocode 4—Design of composite steel and concrete structures—Part 1-2: General rules - Structural fire design," Brussels, Belgium, 2008.

[5] D. Rush, L. Bisby, M. Gillie, A. Jowsey, and B. Lane, "Design of intumescent fire protection for concrete filled structural hollow sections," *Fire Saf. J.*, vol. 67, pp. 13–23, Jul. 2014.

[6] CEN, "BS EN 1993-1-2:2005: Eurocode 3: Design of steel stuctures; Part 1-2: General rules - Structural fire design," Brussels, Belgium, 2009.

[7] D. Rush, "Fire performance of unprotected and protected concrete filled structural hollow sections," University of Edinburgh, 2013.